

INSPECTION AND REPAIR OF THE DETERIORATED RT.72 MANAHAWKIN BAY BRIDGE

Xiaohua H. Cheng¹, Gregory Renman¹ and Richard Dunne¹

Abstract

The superstructure of Rt.72 over Manakawkin Bay Bridge consists of seventeen riveted steel span, continuous, cantilevered girder and floorbeam system with pin/hanger suspended spans. Since the bridge was “Structurally Deficient”, rehabilitation work was carried out in early 1990s, including deck resurfacing, pin/hanger connection replacement, cleaning and repainting the steel structure. During rehabilitation, fatigue cracks and deck haunch damage were found at numerous locations. The fatigue cracks were all similar, that is, horizontal cracks in the floorbeam webs and vertical cracks in the floorbeam-to-girder bracket connection angles. The horizontal cracks have been repaired by hole drilling method and monitored in each following routine inspection.

Over the years, the number and extent of the fatigue damage has been continuously increased since they were first reported. Some cracks further propagated beyond the previously drilled holes. It appears that fatigue cracks and deck haunch spalling were due to out-of-plane bending resulted from relative displacement between the concrete deck and the top flange of the girder as live load passes across the bridge as well as the differential temperature movement.

The paper presents the details of the bridge structures, inspection findings, fatigue damage, cause of the damage and repair of the damage.

Introduction

Currently there are nearly 600,000 bridges in all public highway systems in the United States. Approximately 26% of them are structurally deficient and functionally obsolete (FHWA, 2005). Based on the Year 200 National Bridge Inventory (NBI), the average age of the bridges is 40 years old (48 years old for steel bridges). In contrast, New Jersey has nearly 6,500 highway bridges (2,579 NJDOT bridges). About 36% are structural deficient and functionally obsolete. The average age is 48 years old with steel bridges even older, which is worse than the national condition. Under the sponsorship of federal and state funds, the New Jersey Department of Transportation (NJDOT) is making great efforts to rehabilitate and replace the deteriorated bridges to ensure public safety and movability.

The NJ Route 72 over Manahawkin Bay Bridge was opened to traffic in 1959. It is

¹ Bureau of Structural Engineering, New Jersey Department of Transportation, Trenton, NJ 08625

the only vehicular access to the Long Beach barrier island of Jersey Shore and is particularly important during summer season. The superstructure consists of seventeen span, continuous, cantilevered girder and floorbeam system with pin/hanger suspended spans. The bridge is 2400 feet (732 m) long and 68.8 (21 m) feet wide with four traffic lanes, two sidewalks and a median barrier (**Figure 1**). The maximum span length is 150 feet (46 m). The bridge was originally designed with HS-20 load. The recent ADT is approximately 45,000 and ADTT is 7%. Note that there are five drop-in pin/hanger suspended spans, but the bridge is not classified as “Fracture Critical” because of the four-girder superstructure system.

After 1967 Silver River Bridge collapse, Federal Government mandated the National Bridge Inspection Program (NBIP) and the associated Highway Bridge Replacement and Rehabilitation Program (HBRRP). NJDOT started the bridge inspection program in 1971 when National Bridge Inventory System (NBIS) was established. The Rt.72 Manahawkin Bay Bridge was in the first ones that were inspected. In 1990, the bridge was classified as “Poor” for the overall condition. The overall structural evaluation was rated 4 as “Meet Minimum Tolerate Limits”. At that time, the ADT was about 39, 2000 and ADTT was 7%. The load rating by load factor method was satisfied except interior floorbeams for inventory rating due to the increased dead loads since the original design. The bridge was severely corroded in girders, stringers, floorbeams, wind locks, expansion joints and pin/hanger connections, as shown in **Figure 2**. The pin and hanger assemblies were inspected by ultrasonic testing (UT) and found to be free of any detectable indications. Since the bridge was “Structurally Deficient”, repair and rehabilitation were recommended. For superstructures the following was recommended and carried out to retard further deterioration:

- 1) Resurface the deck with Latex Modified Concrete (LMC);
 - 2) Replace the pin and hanger connections;
 - 3) Clean and paint superstructure steel and bearings;
 - 4) Rehabilitate expansion finger dams (repaint dams with skid-resistant paint), or replace deck expansion joint;
 - 5) Replace sheared rivets with high strength bolts (two spans)
 - 6) Rehabilitate wind locks to working condition
- and so on (median barrier and parapet replacement etc.).

During the ongoing cleaning and painting of the steel superstructure, fatigue cracks were found at numerous locations. They are all similar, that is, horizontal cracks in the floorbeam webs and vertical cracks in the floorbeam-to-girder bracket connection angles. In this bridge, fatigue details were identified for evaluation: Vertical tack welds (AASHTO Category D for 2-4 inch long (50-100mm)) at floorbeam-to-girder bracket angles; Riveted connections (Category D net section); and the out-of-plane bending at the top floorbeam web (distortion at small gap).

This paper will present details of the bridge inspection and repair/rehabilitation, by

focusing on fatigue damage, cause of the damage, and repair and monitoring of the damage and other deteriorated members. In addition, electrochemical fatigue sensors were used to identify which cracks were continuing to propagate beyond previously drill holes. This will also be briefly discussed.

Observations of Fatigue Damage and Deck Haunch Damage

Since fatigue cracks were uncovered during the repair/rehabilitation, NJDOT decided to take measures to test the locations and extent of the floorbeam fatigue damage by using dye penetrant method or magnetic particle method or UT, whichever was proper in the field, at all floorbeam web and bracket angle details. The connection detail is shown in **Figure 3**.

The in-depth inspection was first performed in 3-1/2 end spans (the 1st and 2nd spans at east end of the bridge and 1st and half 2nd span at west end of the bridge). 5 out of 68 floorbeam locations showed horizontal cracks in the floorbeam webs, measuring 7~11-1/2 inch (178~292mm) in length. All 5 locations also showed vertical cracks at the top in the floorbeam-to-girder bracket angles, measuring 1-1/2~6 inch (38~152mm) in length. The typical sketch and photo of the fatigue cracking are shown in **Figure 4**. In addition, concrete deck haunch cracking and spalling was displayed in all spans inspected with several areas extending as much as 6 to 10 feet (1.8 to 3.0m) in length (**Figure 4 and Figure 5**). Note that the corrosion at edge of top flange of floorbeam was due to water penetration through the deck.

In the intermediate spans, 22 of 60 floorbeam locations showed horizontal cracks in the floorbeam web and/or vertical cracks in the bracket angles at the top. Deck haunch spalling was also common throughout the spans. It was also observed that deck joint opening was measured 1~5-1/4 inch (25~133mm) when the inspection was performed in June.

At this time, the recommendation was made that the crack tips in the floorbeam webs be determined, stop holes be drilled at the crack tips and cracks be monitored for any further propagation. Cracks in the bracket angles need be monitored only. It was also recommended that expansion joints be monitored on a regular basis to determine if the bridge is expanding and contracting properly.

Other recommendation was made to disconnect all the brackets by cutting the gusset plates, which might allow the floorbeam to deflect by rotating about the bottom flange of floorbeam, instead of the very short gap between the floorbeam top flange and the bracket angle top, thereby inducing much smaller moments and stresses in the floorbeam web. However, this recommendation was not taken.

Cause of Fatigue Damage and Deck Haunch Damage

It was believed that the horizontal/vertical fatigue cracks and deck haunch spalling were caused by out-of-plane bending of the floorbeam web due to the differential longitudinal movement (or relative displacement) between the deck slab and top flange of the continuous girders as live load passes across the bridge as well as the differential temperature movement.

Before the pin/hanger connections were replaced, some of the connections were corroded and frozen (**Figure 2**), which made girder not move freely at the connection. At the pier locations, the girders are continuous while the deck expansion joints were free to expand and contract with two floorbeams separate on each side of expansion joint. Since the floorbeam-to-girder bracket restrained such free movement at the top near the deck slab, the relative movement in the floorbeam web between the floorbeam top flange and top of bracket angles, that is out-of-plane bending in the floorbeam web, were significant (**Figure 6**) evidenced by the concrete spalling of deck haunch. As a result of the repeated out-of-plane bending, fatigue cracks developed at the floorbeam top flange weld toe. On the other hand, the bracket angles are connected to both the floorbeam and bracket plate with rivets, while the other side of bracket plate connected to the girder. When deck is moving, the angles are subject to repeated pry effect which caused fatigue cracks at the corner of the angles. Since the out-of-plane bending is very complex, further study on more refined analysis for the bridge global behavior and local stresses may be necessary to interpret the observations and provide guidance to avoid the poor structural design and detailing. .

Over the years, the number of fatigue crack locations and the extent of the damage have continuously increased since fatigue damage was first reported in early 1990s. The number of cracks has grown from first 27 in 1993, to 110 in 1995, 150 in 1997, 200 in 1999, until 129 in 2005. Many cracks further propagated beyond the previously drilled holes. The new holes were drilled at new crack tips. **Figure 7** gives a sketch of fatigue crack locations in a recent inspection. It is noted that fatigue cracks have occurred in floorbeams at all four girder locations. It appears that floorbeams at fascia girder locations have more cracks than interior girder locations, and the floorbeams at/near fixed bearing locations and pin/hanger locations most likely developed fatigue cracks.

Repair of Fatigue Damage and Deck Haunch Damage

Basically, all the fatigue cracks in the floorbeam webs have been repaired by drilling holes at the crack tip to stop crack propagation, as seen in **Figure 8a**. Visual monitoring of the cracks has been made during each following routine inspection. The number and extent of fatigue cracks increased over the years. For the newly developed cracks, holes were drilled in the same manner. The typical stop hole diameter was ½ inch (12.7mm). For the cracks that further propagated beyond the previously drilled hole, new holes were drilled at new crack tip. Such repair procedure may be repeated several times as shown in **Figure 8b**, leaving multiple stop holes of various diameters in a floorbeam

web. All the repairs were made at Priority I level.

For the vertical cracks in the floorbeam-to-girder bracket connection angles, they have been monitored in each following routine inspection. If the cracks propagated downwards in the angle significantly, stop hole would be drilled at the crack tip, or the angle would be replaced with new angles and high strength bolts (**Figure 9**).

Over 75% of all haunch area were cracked, fractured and spalled in the inspection of 1997. Repair by patching was called for but no action has been taken so far. Only monitoring is made during each cycle of inspection, considering the satisfactory composite effect (shear studs) between the deck slab and floor beam top flange.

Electrochemical Fatigue Sensor (EFS) Monitoring

The EFS system was used on the Rt.72 Manhawk Bay bridge to help monitor fatigue crack propagation as a pilot study (MTI, 2006). The EFS system (a pair of crack measurement sensor and a reference sensor) was installed at 17 locations, that is, at visually identified cracks, areas adjacent to crack repair locations, and additional locations that exhibited similar fatigue-sensitive details as those already cracked. The EFS data analysis was used to determine whether the documented fatigue cracks were still growing under ambient traffic flow and the possible inspection of cracks that were not detected by dye penetrant and magnetic particle methods. The testing of EFS system showed that some of existing cracks were continuing to grow and others had been arrested by stop-holes and were not exhibiting active growth. The technology was confirmed by this test. **Figure 10** shows the EFS system installed at existing crack that propagated beyond a stop-hole. Further details can be obtained in the MTI report.

Summary

The original structural design of the bridge did not consider fatigue and the out-of-plane bending was not taken into account. The bracket connection detail restrained the floorbeam movement, and thus the inconsistency of deck joint movement and girder movement jointly caused the out of plane bending in the floorbeam web. At the poor detail (small gap between floorbeam top flange and floorbeam-to-girder bracket angles), fatigue cracking and deck haunch spalling developed. However, the damage is not thought to be fracture critical due to four-girder system redundancy. Retrofit of such details and the deck joint system is very difficult, therefore monitoring of the damage has been performed during biannual routine inspections. Fatigue Cracks were repaired by hole drilling. Once further damage was found, new repair was carried out, and regular monitoring is made for safety. There is difficulty arresting the crack propagation using stop holes because the observation shows more and more cracks developed and propagated.

To find out the mechanism of out-of-plane behavior and the fatigue and deck haunch

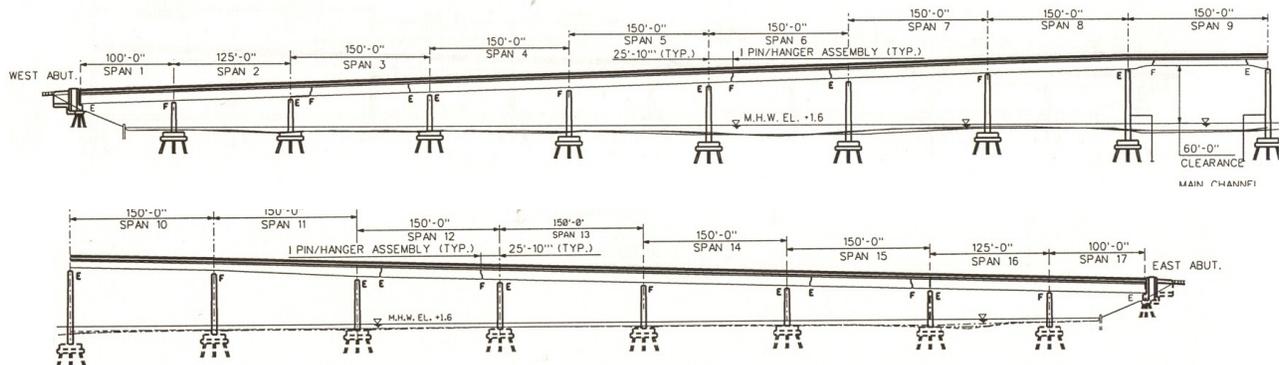
damages, further detailed structural analysis for global structural behavior and local stresses is needed. Although the bridge is still in use safely, the original design of poor structural configuration and detailing made the bridge maintenance a great burden to the bridge owner. The lessons have to be learned. Since the bridge is close to the design life, the plan of major rehabilitation/retrofit or entire bridge replacement is under consideration and evaluation.

References

Federal Highway Administration (FHWA), “Status of the National’s Highways, Bridges, and Transit: 2004 Conditions and Performance, Chapter 15 – Bridges”, 2005

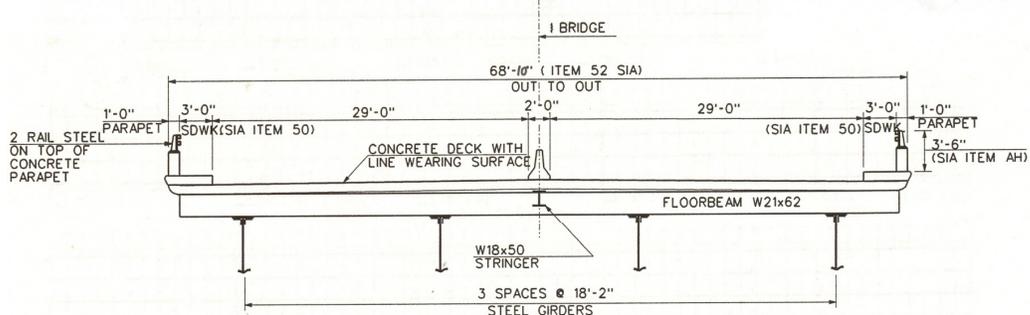
New Jersey DOT, “bridge Re-Evaluation Survey Report of the Route NJ 72 Over Manakawkin Bay”, 1990, 1993, 1995, 1997, 1999, 2001, 2003, 2005

Material Technologies, Inc. (MTI), “Active Fatigue Crack Inspection on the New Jersey Department of Transportation Manahawkin Bay Bridge Using the Electrochemical fatigue Sensor”, Dec., 2006



ELEVATION

1" = 100'-0"



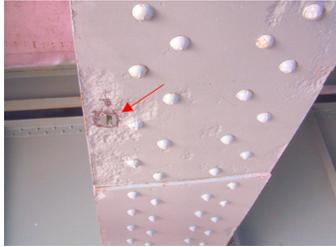
TYPICAL CROSS SECTION A-A

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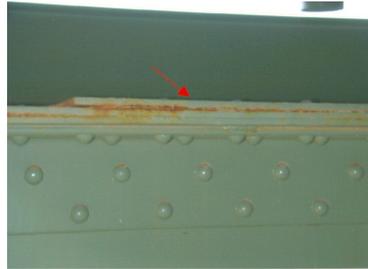


General View of Superstructure

Figure 1 Elevation, Cross Section and Superstructure of the Bridge



In Main Girder Flange



Between Girder Flange Cover Plates



In Floorbeam Web



In Pin/Hanger Connection



Under Expansion Joint



In Wind Lock

Figure 2 Corrosion of Various Bridge Members

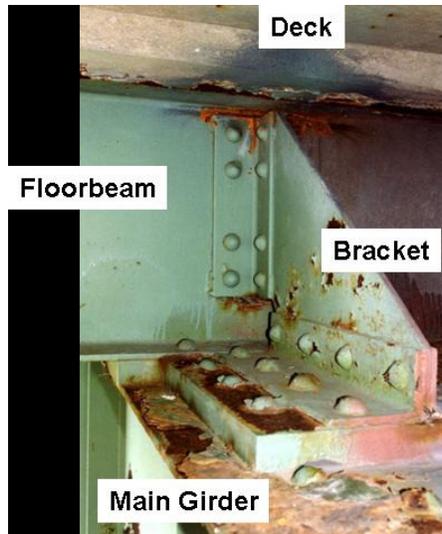


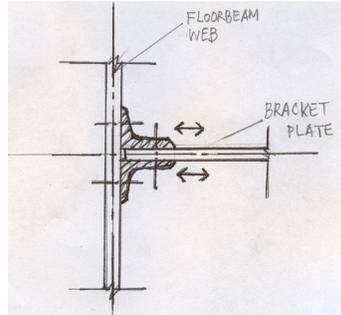
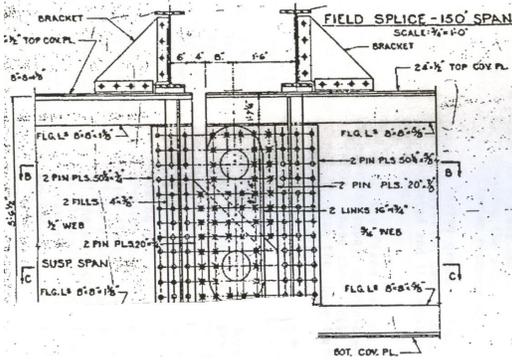
Figure 3 Floor-to-Girder Bracket Detail



Figure 4 Fatigue Cracks - Horizontal Crack in Floorbeam Web and Vertical Crack in Bracket Angle



Figure 5 Concrete Deck Haunch Spalling



Floorbeam-Bracket-Girder Detail

Pry Effect of Angle Causing Cracking

Figure 6 Relative Movement Between Deck and Main Girder

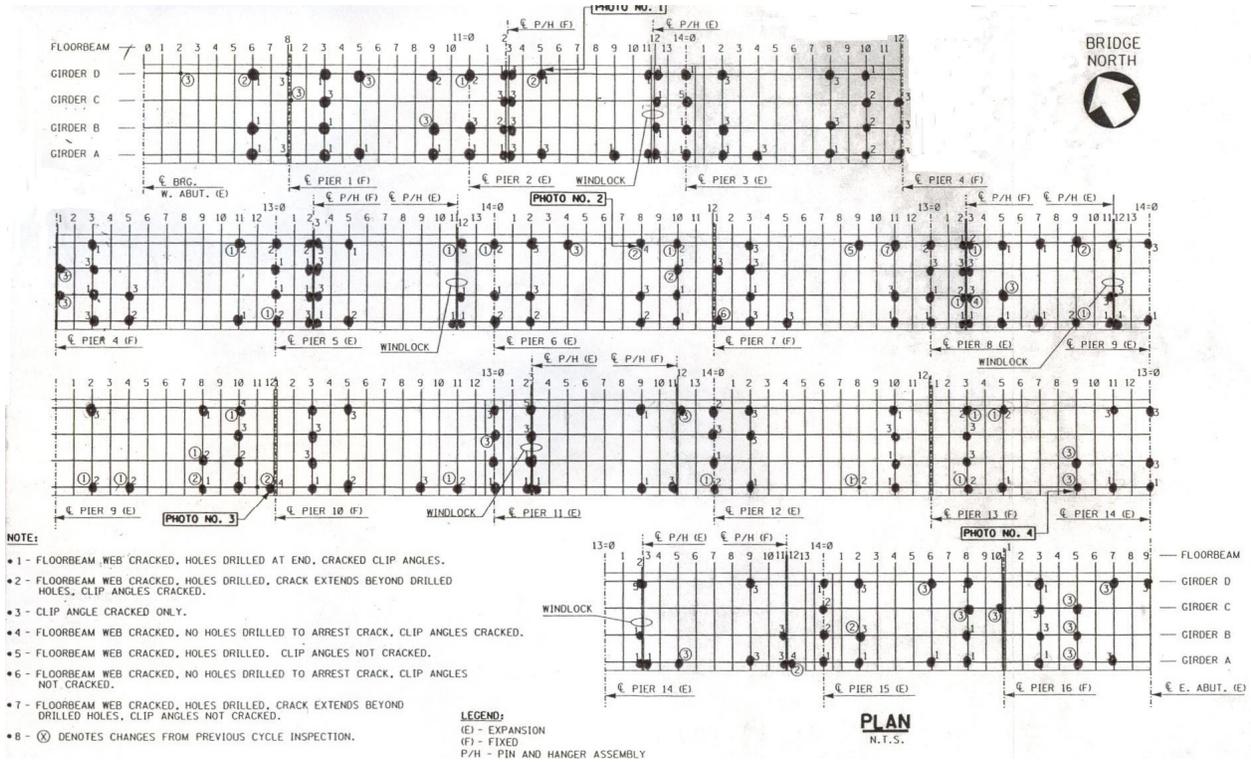
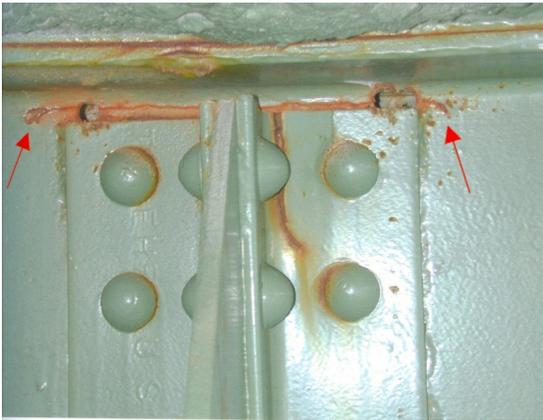


Figure 7 Fatigue Crack Locations in the Bridge



(a) First Repair



(b) Multiple Repairs

Figure 8 Hole Drilling Repair of Fatigue Cracks



(a) Hole Drilling



(b) Angle Replacement

Figure 9 Repair of Fatigue Cracks in Angles



Figure 10 EFS System Installed At An Existing Fatigue Crack